DETAILING

OF

STRUCTURAL STEEL

FOR

OFFICE BUILDINGS

BY

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BERKELEY AND APPLETON STREETS

BOSTON

PREFACE.

THESE notes have been arranged for the students in *Structures A* at Franklin Union. They are to be used in connection with the author's class room lectures, and are intended to cover only the elements of the subject.

For a more complete discussion of the subject, the student is referred to the following books:—

Shop Hints for Structural Draftsmen—John C. Moses.
Bridge Design, Vol. III—Merriman and Jacoby.
Steel Mill Buildings—Milo S. Ketchum.
Structural Design—W. Chase Thompson.
Steel Structures—Clyde T. Morris.

DETAILING OF
STRUCTURAL STEEL FOR OFFICE BUILDINGS.

Foundation Plan.

The first thing generally required in a drafting room is the Foundation Plan. The foundation plan should show the location and thickness of all walls, the location, grades and sizes of column bases and which way they set; it should also show any beams used for footings, etc., and in general, should give all the information needed by the mason to enable him to have everything ready for the erection of the steel, and should serve as an erection plan for the cast bases, grillage beams, etc. Column Schedule and Framing Plans of the different floors are next made.

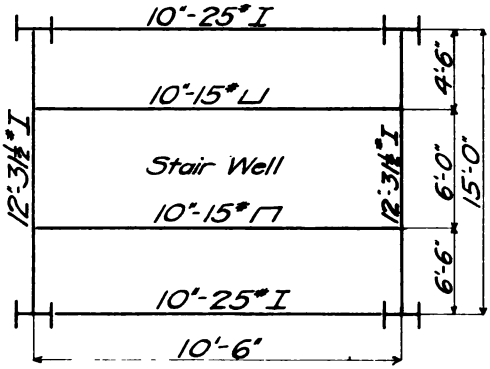
Column Schedule.

The Column Schedule should show the size and makeup of all columns, the total column load in each story, the grades of all finished floors, column splices, and tops of column bases, stating whether the bases are of cast iron, steel plates or grillage.

Framing Plans.

Framing Plans should show location and thickness of all walls, size and spacing of all **I**'s and **C**'s, lintels, if any, that are over windows in the story below, wall plates and sizes of same, and should give all dimensions and grades necessary for making the detail drawings. (On framing plans show the walls that are on the architect's plan of the floor below). Dimensions are given to the center of columns, center of beam webs, and to the back of channels.

Framing Plans must show which way columns and channels are turned, thus:—



This sketch indicates that the 10″ **C**'s are 6′–0″ back to back, and are turned with their flanges away from the stair well.

All erection numbers for beams and columns should be plainly given. All *beams* on the same floor that are alike should be given the *same mark*. *Columns* are generally given *different numbers*, even though they are alike.

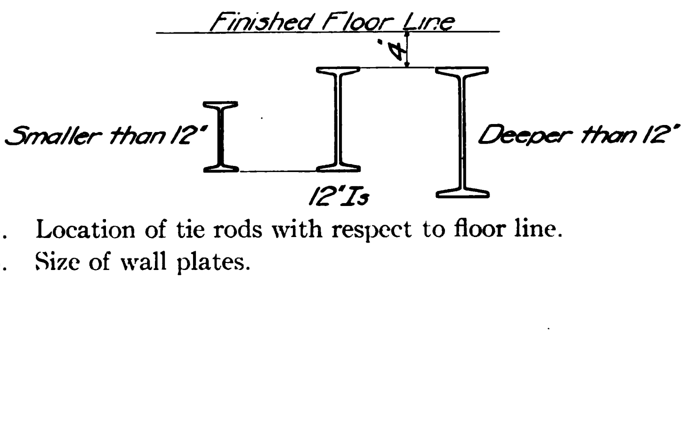
C's at walls are usually placed 1″ clear of wall with flanges turned away from wall.

If the floor construction requires the use of tie rods, they should be shown by dotted lines. They are generally ¾″ diameter, stagger 3″, are about 2½″ longer than the distance *c.* to *c.* of **I**'s, and have 2½″ thread and one nut at each end.

Each Framing Plan of a building should have the following notes in a group:—

1. Grade of finished floor.
2. Distance from finished floor to tops of main beams.
3. Relative grades of beams.

If all beams are flush on top or on bottom a note will state the fact; if not, a sketch should be made similar to the following:—



4. Location of tie rods with respect to floor line.
5. Size of wall plates.

6. Floor letter.
(A is generally used for 1st floor; B for 2d floor, C for 3d floor, etc.)
7. Grade of column splices.
(Generally 12″ above finished floor line.)
8. State if wall anchors are to be used.
9. Give street names or points of compass.

Plate and Angle Columns.

Columns are generally made in two-story lengths, with splices 12″ above floor line and all at the same grade. Cast Iron Bases are generally used for columns, and steel wall plates for beams. The bases may be set to grade before columns are shipped and the columns are bolted to them afterwards.

Plate and angle columns are generally made ¼″ wider back to back of **L**'s than the width of web plate.

Columns are generally faced at both ends. In general, splice plates are not depended upon to carry load, and are made of arbitrary thickness, (say ¼ to ⅜ inch).

Splice plates on flange of columns are made full width of column. Those on web should be ½″ narrower than the clear distance between flange **L**'s, as **L**'s may overrun in width.

Splice plates should be bolted or riveted to lower section of columns, with rivets nearest the end of column left out to allow splice plates to spread, so that the upper section of column will enter easily.

Beams framing to *flange* of columns may be fastened with connection **L**'s on the web of beam or may rest on a seat **L**. The seat connection is preferable as it saves shop riveting on beams, thus allowing them to be taken to the shipping yard from the shop as soon as they are punched. Beams framing into *web* of column should rest on a seat **L**.

In the very high buildings of New York and Chicago, the beam connections are made rigid to resist wind pressure; but on smaller buildings this is neglected.

In detailing columns, one line of dimensions should locate all rivets in web; one line, only those in flange; one line, seats and clips on web; another, those on flange. One line should give faced length; and another, the distance between floor lines. (See typical column detail).

Beam Seats on Columns.

A beam seat must be connected to the column by a sufficient number of rivets to carry the reaction from the beam. In the case of a beam uniformly loaded, the reaction will be one-half the safe load for the beam as given in the hand book. If the beam is not uniformly loaded, the reactions should be figured.

If a beam seat requires 4 rivets or less to carry load, make seat **L** 6″ x 3½″ x ½″ with 6″ leg on column and use 2¼″ gage in 3½″ leg.

Use 3½″ x 3″ x ⅜″ clip angle above beam with 2¼″ gage in outstanding 3½″ leg and 1¾″ gage in 3″ vertical leg. Ship clip **L** bolted to column allowing ¼″ clearance above **I**.

In case beam is on a skew or is very heavy, seat **L** may have to be 6″ x 4″ or even 6″ x 6″.

For double **I**'s generally use 6″ x 6″ seat **L**'s and rivet the **I**'s thru their outside flanges only.

Beam Seats with Stiffeners.

If more than four rivets are required in seat **L** to carry beam load, Stiffener **L**'s should be used with their top ends fitted against outstanding leg of seat **L**.

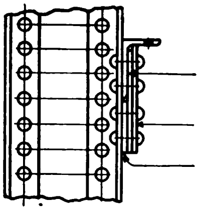
If stiffeners are used, make seat **L** 6″ x 3½″ x ⅜″.

The outstanding leg of stiffener **L**'s will generally be 3″ for 3½″ seat **L**, 3½″ for 4″, and 5″ for 6″, respectively.

Deducting ½″ which will be cut away to clear fillet of seat **L**, the outstanding leg of stiffener should be thick enough to carry load, allowing 18,000 or 20,000 pounds per square inch on remaining area. Rivets in the leg of stiffener **L** which fastens to column are put on the gage line of column. This, together with the necessity for having outstanding leg clear rivet holes in seat **L**, will determine size of the riveted leg; which should be as small as possible, to avoid eccentric loading. If a large gage is necessary, rivets in the stiffener **L** should be figured for the eccentric load.

If stiffener **L**'s cannot be used because of lack of room and seat **L** contains enough rivets to carry the load, a filler, say ⅜″ thick, is sometimes riveted between seat **L** and column to insure beam bearing on vertical leg of seat angle. The filler should be large enough to get two rivets into column besides those thru the seat **L**.

Another method when stiffener **L**'s cannot be used is to substitute a plate for the stiffener **L**'s thus:—



Filler same thickness as radius of fillet of seat **L**.

Plate fitted at top to take load.

Filler same thickness as **L**.

Bottom ends of stiffener **L**'s under seats on *flange* of columns are generally cut at 45°. Those on *web* of columns may be cut square.

Seat **L**'s and clip **L**'s on web of column must clear fillet of column **L**. Cut ends square if sufficient lap can be obtained; if not, they should be fitted to column **L**'s.

When possible, connections should be so arranged that all web rivets in columns can be driven first, and flange rivets afterward. Should it be impossible to drive some flange rivets afterward, because of interference with stiffener **L**'s, as would be the case with seat for the 20″ **I** shown on typical column detail, the stiffeners will not be riveted until after the seat **L** on flange. Sometimes it may be impossible to arrange connections so that all rivets can be driven by machine, *but hand-driven rivets should be avoided if possible.*

Some engineers call for stiffeners in pairs to have their outstanding legs riveted together. When stiffeners are on flange of column this can be easily done after they are riveted to column. (See typical column detail.) But if stiffener **L**'s are on the web of column, there is not room enough to drive good rivets after they are riveted to column; and if they are riveted together in pairs before being riveted to column, there is a good chance that top ends will not be even, (unless they are faced, which is expensive), and that one of them will take all of the load.

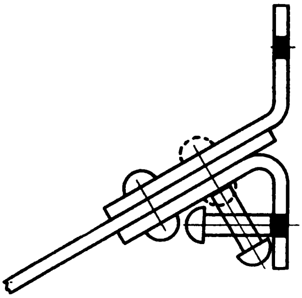
Beam Connections.

For beams and channels that frame into webs of other beams, use Standard connection angles shown in the Handbook. (See Carnegie or Cambria for table of minimum spans for **I**'s fully loaded for which standard connection **L**'s may be used.)

These angles are generally located in the centre of webs of **I**'s, except when beams of different depths framing opposite are either flush on top or bottom, in which case they should be located as shown in table in handbook.

Allow $\frac{1}{16}''$ clearance at each end of beams with connection **L**'s on web. In other words, make the distance back to back of **L**'s $\frac{1}{8}''$ less than the clear span.

In case beams frame into other beams at a small skew, standard connection **L**'s may be bent and used. If the skew is considerable, a 3″ scale or full size layout must be made, and if necessary the gages in both legs of the **L**'s must be increased to allow rivets to enter the holes. Bent plates are sometimes used instead of **L**'s.

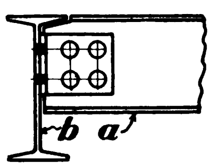


I's are ordered say $\frac{1}{2}''$ to $\frac{3}{4}''$ short, to allow for variation in cutting to length at the mills.

If **I**'s rest on a seat at column, allow $\frac{1}{4}''$ clearance between face of column and end of **I**, and note ($+^{0''}_{\frac{1}{4}}$) on detail.

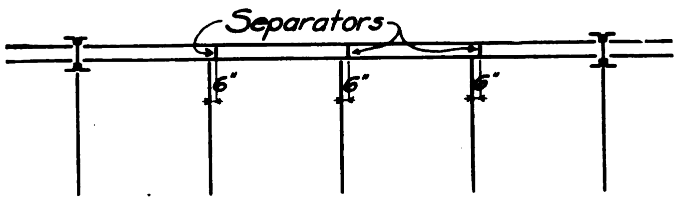
Coped Beams.

When two beams are framed flush on top or on bottom as shown in the sketch, a portion of both web and flange of "*a*" is cut to clear the flange of "*b*". This method of cutting is called coping, and a beam so cut is called a coped beam.

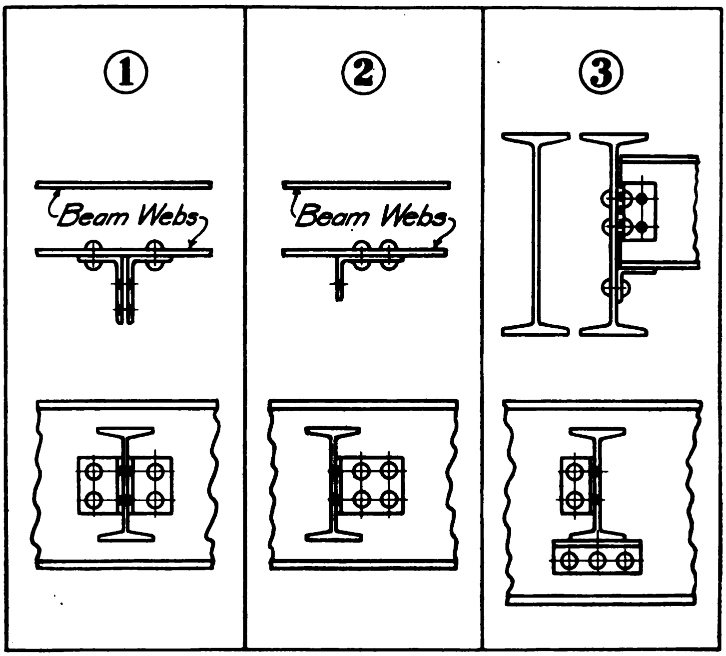


Beams with Separators.

When double beams are used, they are shipped bolted together with standard separators spaced 5 or 6 feet apart. (See hand book). In case of **I**'s framing into one of a pair of double **I**'s, separators should be located as near as possible to the connections—say 6″. Separators so placed help to distribute the load equally over both of the double **I**'s.



Standard connection **L**'s should not be shop riveted to webs of beams which frame into double **I**'s, as the field rivets cannot be driven. Connections may be arranged in any of the following ways:—

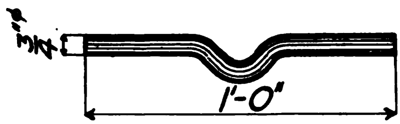


If standard connection **L**'s are shop riveted to one of a pair of double **I**'s as in sketch (1), the clear distance between them should be $\frac{1}{16}''$ more than the thickness of the web that frames in. One objection to this method is the difficulty in erection; and in no case should both ends of a beam be so connected.

Sketch (2) shows a single **L** shop-riveted to one of the double **I**'s. The outstanding leg will carry only shear from beam; but rivets in the other leg must be figured for the eccentric load as well as direct shear. (See pages 16 and 17.)

Sketch (3) shows a good detail to use when double **I**'s are deep enough to allow it. The seat **L** should carry all the load, the side **L** being used only for stiffness.

Wall Anchors.



Three or four different kinds of anchors are used to fasten beams to a wall. Perhaps the most common is a bar ¾" diameter, bent as shown, which slips into a hole in web 2" from end of beam. For anchors for double **I**'s use a straight ¾" bar, say 1′–6″ or 2′–0″ long.

General Notes on Making Drawings.

Detail drawings are made to a scale of ¾" or 1" to the foot.

For small and intricate work, castings, etc., 1½" or even 3" to the foot may be used.

Framing plans may be made to a scale of ⅛" or ¼".

Use dull side of tracing cloth, and use fine lines for dimensions (full lines, not dotted) and heavy black lines for steel.

Red ink should *only* be used to show parts of a structure that are detailed on another sheet, and connect to portions shown; and this is necessary *only* for *complicated* work.

Beams are generally all drawn the same length, with details not necessarily to scale. Most bridge companies have printed blanks for beam details.

Columns should be drawn to scale *except length.* It is well to show web seats dotted in flange views, as by so doing, interferences will be more easily detected by the draftsman.

In writing size of **L**'s the long leg should be given first, then the short leg, then the thickness, and last the length in feet and inches. Thus:—2 **L**'s 6″ x 3½″ x ⅜″ x 4′ -2½″.

In writing the size of plates, first give the width in inches, then the thickness, and last the length in feet and inches. Thus:— 1 Pl. 12″ x ⅜″ x 1′-6½″.

In detailing columns, the size of material and the shop marks may be written on the detail; or shop marks *only* may appear, and a list showing sizes and shop marks of all material for the column may be given in a table *near* the detail. See typical column detail.

The following is a system of shop marks used in some offices:—

Splice Plates are marked . . .	B1–B2–B3, etc.
Diagonals	D1–D2–D3, etc.
Filler Plates	F1–F2–F3, etc.
Gusset Plate	G1, etc.
Knee or Connection **L**	K1, etc.
Shoe Plate	SP1, etc.
Tie Plate	T1, etc.
Upright	U1, etc.
Washer	W1, etc.

All pieces that are alike are given the same mark.

Avoid eighths and smaller fractions of inches when practicable.

Tie in rivet spacing at every opportunity. When a number of equal spaces are given, always give the total distance. Thus:— 6 @ 3½″=1′ -9″.

Give the total distance between holes in outstanding legs of end connection **L**'s, also give the gage in **L**. If the two legs of a connection **L** are alike except for a slight difference in gage, change the gage in the shop-driven leg to be the same as the other leg, to avoid error in shop assembly.

Notes on Size of Rivets.

For building work generally use ¾″ rivets. If any 2″ **L**'s are used, rivets must be ⅝″. It is often cheaper however, to increase a few 2″ **L**'s to 2½″, so that ¾″ rivets may be used throughout the job. It costs no more to drive a ¾″ rivet than a

5/8" rivet, and fewer will be required. Rivets larger than 3/4" will not be required on buildings, except for very heavy work. If possible *use only one size of rivets* on a job. If more than one size is used, it means extra work in carrying material from one punch to another, or changing punches in the machine, and increases the chance of error in shop. A few holes are very sure to be punched wrong size, and then the smaller must be reamed out in one piece, to match the larger ones in the piece to which it connects. If only one size rivets are used throughout a job, it also simplifies the erection, as fewer tools are required. However, if more than one size *must* be used, make the change in the small pieces that are easy to handle in the shop.

Notes on Spacing of Rivets.

Minimum Pitch (or distance between centres of rivets) should not be less than three diameters of the rivets; or for

5/8" rivets, 1 7/8"
3/4" rivets, 2 1/4"
7/8" rivets, 2 5/8"

Maximum Pitch of rivets in column webs should not be more than sixteen times the thickness of the flange angles, nor exceed 6 inches.

Rivet pitch at ends of columns and other compression members should not exceed 4 times the size of rivet for a distance equal to twice the width of the member. In column webs close spacing should be used near all beam connections.

Minimum Lap (or distance from centre of a rivet to edge of a plate or angle) should not be less than 1 1/2 times the diameter of the rivet, and should be twice the diameter when possible.

Lap for 5/8" rivets generally 1 1/4"
3/4" rivets generally 1 1/4" or 1 1/2"
7/8" rivets generally 1 1/2" or 1 3/4"

In some cases it may be necessary to reduce these figures slightly, but they should be adhered to when possible.

Rivet holes are punched 1/16" larger than the diameter of the rivets.

Thickness of metal should not exceed the diameter of punched rivet holes, as there is danger of breaking the punches.

Steel Bearing Plates for I Beams.

Where the end of a beam rests on a wall, a bearing plate will generally be required.

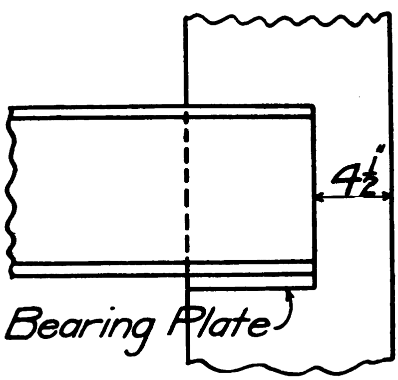
This plate must have sufficient area and thickness to properly distribute the load from the **I** or **C** over the wall.

Both steel and cast iron are used for bearing plates; but, as steel is much to be preferred, only *steel plates* will be considered here.

Tables of Wall Bearing and size of Bearing Plates for different depth of **I**'s are given in Carnegie Handbook. These are frequently larger than necessary.

It is difficult to make a general rule for length of wall bearing, as this depends upon both the size and length of the **I**. In general however, we may say that it should be ⅔ the depth of **I** when possible, never less than half the depth, nor less than 6″, except for 3″ or 4″ **I**'s used as lintels.

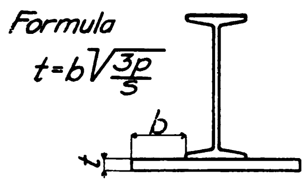
If convenient allow at least 4½″ from end of **I** to outside face of wall. Thus:—



It is considered good practice to allow a load of 150 pounds per square inch on brick laid in lime mortar, and 200 pounds on brick in cement mortar.

Dividing the total load on shoe plate by allowable load per square inch on brick or masonry, will give the required area of plate.

The thickness may be obtained from the following formula:—

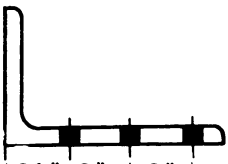


t =thickness of plate (in inches.)
b=projection of plate beyond beam (in inches.)
p=allowable pressure on wall (in pounds per sq. in.)
s =allowable fibre stress on plate (in pounds per sq. in.)

Let s=16000. Then "*t*" may be obtained for various pressures by multiplying "*b*" by the following co-efficients:—

Pressure in lbs. per sq. in.	100	150	200	250	300	350	400
Coefficient	0.137	0.168	0.194	0.216	0.237	0.256	0.274

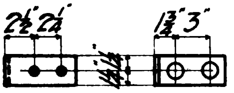
STANDARD GAGES FOR PUNCHING ANGLES.



Size of Leg					
8″	2½″	2″	2″		
	3″	3″			
7″	2½″	3″		May be reversed	Choice of gage may depend upon size of rivets, pitch of rivets and thickness of L's.
6″	2½″	2″			
	2½″	2¼″			
	2¼″	2¼″			
5″	2″	1¾″		May be reversed	
	2″	1⅝″			
4″ *	2½″			These gages are generally used, but may be varied, if desired, to meet special requirements.	
3½″*	2″				
3″	1¾″				
2½″	1⅜″				
2″	1⅛″				

*NOTE.—Stiffener L's and Bracing L's may be punched on the center line.

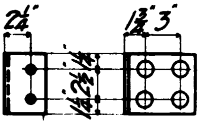
SINGLE ANGLE BEAM CONNECTIONS.



L 6″ x 6″ x $\frac{3}{8}$″ x $2\frac{1}{2}$″

Safe Load 2130#

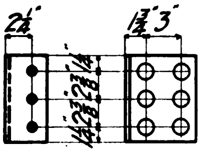
Max. Stress { Shop Rivet 3370
Field Rivet 1065



L 6″ x $3\frac{1}{2}$″ x $\frac{3}{8}$″ x 5″

Safe Load 6740#

Max. Stress { Shop Rivet 4240
Field Rivet 3370

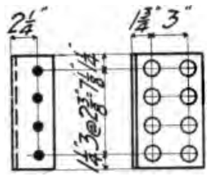


L 6″ x $3\frac{1}{2}$″ x $\frac{3}{8}$″ x $7\frac{1}{4}$″

Safe Load 12000#

Max. Stress { Shop Rivet 4420
Field Rivet 4000

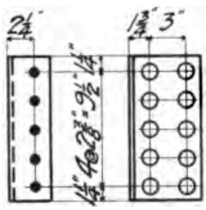
SINGLE ANGLE BEAM CONNECTIONS.



L 6" x 3½" x ⅜" x 9⅝"

SAFE LOAD 17680#

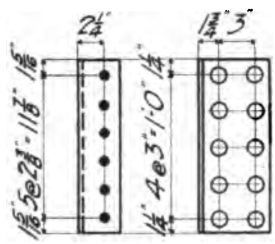
Max. Stress { **Shop Rivet 4370**, **Field Rivet 4420** }



L 6" x 3½" x ⅜" x 1'-0"

SAFE LOAD 22100#

Max. Stress { **Shop Rivet 3920**, **Field Rivet 4420** }



L 6" x 3½" x ⅜" x 1'-2½"

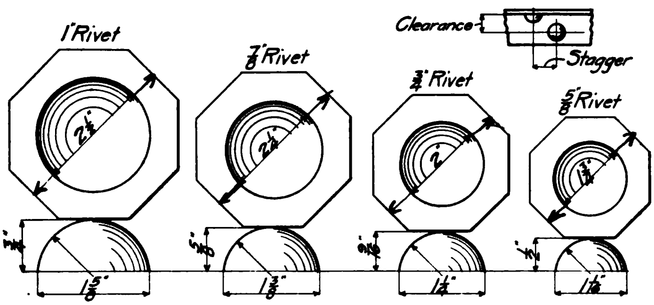
SAFE LOAD 26520#

Max. Stress { **Shop Rivet 4160**, **Field Rivet 4420** }

MINIMUM STAGGER FOR DRIVING RIVETS.

Clearance	1 in.	$1\frac{1}{16}$ in.	$1\frac{1}{8}$ in.	$1\frac{3}{16}$ in.	$1\frac{1}{4}$ in.	$1\frac{5}{16}$ in.	$1\frac{3}{8}$ in.	$1\frac{7}{16}$ in.	$1\frac{1}{2}$ in.	$1\frac{9}{16}$ in.	$1\frac{5}{8}$ in.	$1\frac{11}{16}$ in.	$1\frac{3}{4}$ in.	$1\frac{13}{16}$ in.	$1\frac{7}{8}$ in.	$1\frac{15}{16}$ in.	2 in.
Size of Rivets 1″							$1\frac{1}{2}$	$1\frac{7}{16}$	$1\frac{3}{8}$	$1\frac{5}{16}$	$1\frac{1}{4}$	$1\frac{3}{16}$	1	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{1}{2}$	0
$\frac{7}{8}$″				$1\frac{3}{8}$	$1\frac{1}{4}$	$1\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{1}{16}$	1	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{1}{2}$	0				
$\frac{3}{4}$″		$1\frac{3}{16}$	$1\frac{1}{8}$	$1\frac{1}{16}$	1	$\frac{15}{16}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{7}{16}$	0							
$\frac{5}{8}$″	$\frac{15}{16}$	$\frac{7}{8}$	$\frac{13}{16}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{5}{16}$	0										

DIMENSIONS OF RIVET HEADS AND DIES.



Strength of Riveted Connections.

The strength of a riveted connection depends upon the number and size of the rivets and the thickness of the plates or other pieces which they connect.

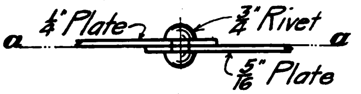


FIG. 1.

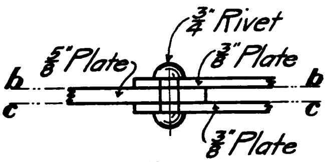


FIG. 2.

If two plates, connected by a single rivet as shown in Fig. 1, are tested to destruction, failure may occur either by the rivet tearing out in one of the plates, or by the rivet being sheared on the line *a–a*. This rivet is said to be in single shear.

If three plates are connected as shown in Fig. 2, failure may occur by the rivet tearing out in the middle plate, or in the two outside plates, or by the rivet being sheared on the lines *b-b* and *c-c*. This rivet will be in double shear and, as regards shearing only, will have twice the value of a rivet in single shear.

The shearing strength of a rivet is equal to the area of its cross section multiplied by the allowable shear per square inch. This allowable shear per square inch varies on different classes of work. It is frequently specified as ¾ the allowable tension unit stress. Thus if the tension unit stress is taken as 16,000 pounds per square inch, the allowable shear will be 12,000 pounds.

The bearing value of a plate is equal to its thickness multiplied by the diameter of the rivet and the allowable bearing value per square inch. This allowable bearing value is generally specified as twice the allowable shearing value. Thus, if the allowable shearing value is 12,000 pounds, the bearing value will be 24,000 pounds per square inch.

Field-driven rivets are not considered as strong as shop-driven rivets, and their value is generally taken at 75% or 80% of the value of shop-driven rivets.

The Boston building laws specify

10,000 pounds per square inch for shear on rivets
and 18,000 pounds per square inch for bearing on plates,

and make no distinction between the value of shop-driven and field-driven rivets.

As an illustration, let us consider the strength of the connection shown in Fig. 1, using the Boston building law values. A $\frac{3}{4}''$ rivet has a cross section of 0.442 square inch, and its value in single shear will be 4420 pounds. The bearing value of the $\frac{1}{4}''$ plate will be $\frac{1}{4} \times \frac{3}{4} \times 18000=3375$ pounds; and for the $\frac{5}{16}''$ plate, it will be $\frac{5}{16} \times \frac{3}{4} \times 18000=4420$ pounds. Failure of this connection would occur by the rivet tearing out in the $\frac{1}{4}''$ plate, and the allowable strength of the connection will be 3375 pounds.

Let us next consider the strength of the connection shown in Fig. 2, using values of 10,000 pounds per square inch for shear and 20,000 pounds per square inch for bearing. The value of the $\frac{3}{4}''$ rivet in double shear will be 8840 pounds. The bearing value of the $\frac{5}{8}''$ plate will be $\frac{5}{8} \times \frac{3}{4} \times 20{,}000=9375$, and for the two $\frac{3}{8}$ plates the bearing value will be $2 \times \frac{3}{8} \times \frac{3}{4} \times 20{,}000=$ 11,250 pounds.

Failure of this connection would occur by double shearing of the rivet, and the allowable strength of the connection will be 8840 pounds. If a stress of 40,000 pounds were to be transferred from the $\frac{5}{8}''$ plate to the two $\frac{3}{8}''$ plates, the number of rivets required would be $40{,}000 \div 8840=5$ rivets.

Tables of shearing value of rivets and bearing value of riveted plates are given in the Cambria and Carnegie handbooks.

In the previous discussion, bending on rivets and friction between riveted plates have been neglected in accordance with common practice.

Bottom Chord Splice for a Roof Truss.

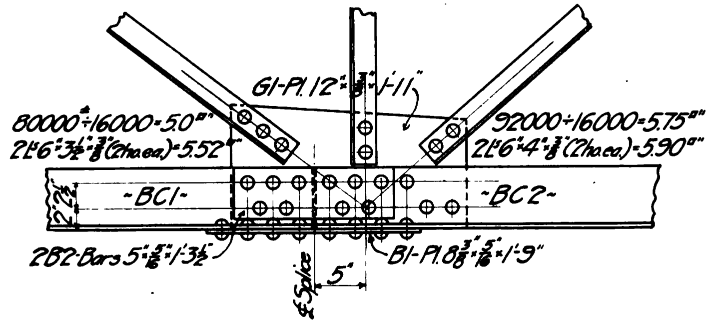
In the discussion of a splice in the bottom chord of an ordinary roof truss of the Warren type given below, the following values have been assumed:—

Tension—16,000 pounds per square inch.

Shear on rivets—10,000 pounds per square inch.

Bearing—20,000 pounds per square inch.

$\frac{3}{4}''$ Rivets will be used; but, in obtaining net areas, the diameter of the rivet holes to be deducted will be assumed as $\frac{7}{8}''$.



Let us first consider the stress in BC1. This stress of 80,000 pounds is assumed to be divided between the two legs of the angles, approximately in proportion to their size. The 3½" legs 7/19 will carry $\frac{7}{16}$ of 80,000=about 30,000 pounds, and the 6" legs will carry the remaining 50,000 pounds.

Splice plate B1 must have a net area of 30,000÷16,000=1.88 square inches. A plate 8⅜" x $\frac{5}{16}$" has a net area of 2.07 square inches and will be used. The rivets which connect this plate to BC1 and BC2 are in single shear, and the number required on each side of the splice will be 30,000÷4420=7 rivets; but as there must be an even number, 8 will be used.

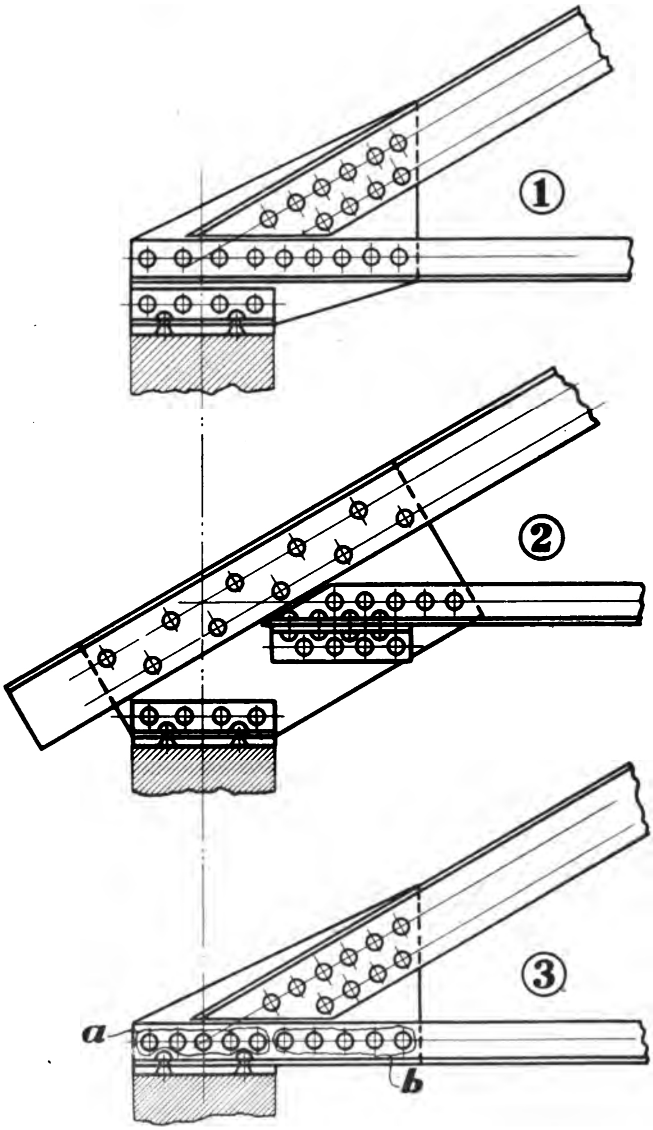
The two splice bars B2 and that portion of plate G1 that is between the vertical legs of BC1 angles should have a total net area of 50,000÷16,000=3.12 square inches. If G1 is made ⅜" thick, its lower portion will have a net area of (6 x ⅜)—(2 x ⅞ x ⅜)=1.59 square inches. The splice bars B2 should then have a total net area of 3.12—1.59=1.53 square inches. The net area of two bars 5" x $\frac{5}{16}$"=2.03 square inches, and they will be used.

The rivets which carry the 50,000 pounds from BC1 to B2 and G1 are bearing on two ⅜" **L**'s, and the number required on each side of the splice will be 50,000÷11,250=5 rivets.

We have now transferred the total stress of 80,000 pounds from BC1 to BC2, where it becomes a part of the 92,000 pounds stress in BC2.

The difference of 12,000 between the stresses in BC1 and BC2 is brought into BC2 from the plate G1 thru additional rivets. These rivets are in ⅜" bearing, and the number required will be 12,000÷2625=3 rivets.

WALL ENDS OF FINK TRUSSES.



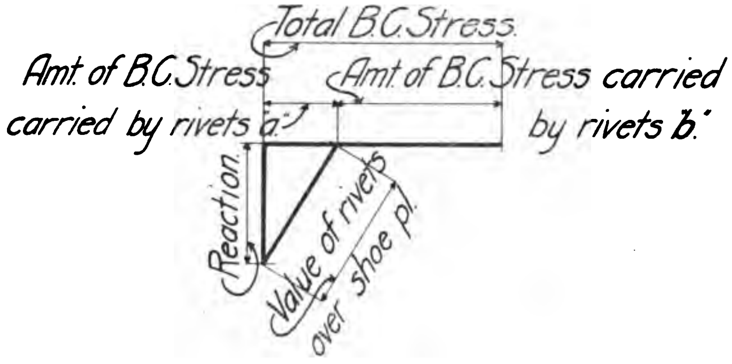
Wall Ends of Fink Trusses.

Three methods of detailing wall ends of Fink Trusses are shown on page 22.

Fig. 1. shows a pair of knees over the shoe plate with sufficient rivets in their vertical legs to carry the truss reaction. The rivets in the bottom chord **L**'s carry only the bottom chord stress.

Fig. 2. shows a similar detail, except that the top chord **L**'s are made longer, to support an eave purlin lower than would be possible with detail in Fig. 1.

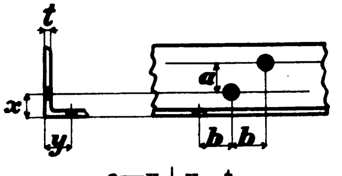
Fig. 3. shows the more common detail in which the bottom chord **L**'s rest directly on shoe plate. In this case, the rivets in bottom chord **L**'s that are over the shoe plate, receive a resultant stress from the combined horizontal bottom chord stress and the vertical reaction from wall. The diagram shows a method of determining the number of rivets. This diagram may be made to any convenient scale.



Note.—The gage lines of top and bottom chord **L**'s should intersect at a point over the centre of shoe plate, as shown in *each* of the above figures; and NOT at *a point outside the shoe plate,* as is sometimes done.

STAGGER OF LAST HOLE IN TENSION MEMBERS

Dist. "a"	Distance "b"			
	$\frac{1}{2}''$ Riv.	$\frac{5}{8}''$ Riv.	$\frac{3}{4}''$ Riv.	$\frac{7}{8}''$ Riv.
$1\frac{1}{2}''$	2 "	$2\frac{1}{4}''$	$2\frac{1}{4}''$	$2\frac{1}{2}''$
2 "	$2\frac{1}{4}''$	$2\frac{1}{2}''$	$2\frac{1}{2}''$	$2\frac{3}{4}''$
$2\frac{1}{2}''$	$2\frac{1}{2}''$	$2\frac{3}{4}''$	3 "	3 "
3 "	$2\frac{3}{4}''$	3 "	$3\frac{1}{4}''$	$3\frac{1}{2}''$
$3\frac{1}{2}''$	$3\frac{1}{4}''$	$3\frac{1}{4}''$	$3\frac{1}{2}''$	$3\frac{3}{4}''$
4 "	$3\frac{1}{2}''$	$3\frac{3}{4}''$	$3\frac{3}{4}''$	4 "
$4\frac{1}{2}''$	$3\frac{3}{4}''$	4 "	$4\frac{1}{4}''$	$4\frac{1}{4}''$
5 "	4 "	$4\frac{1}{4}''$	$4\frac{1}{2}''$	$4\frac{1}{2}''$



a=x+y—t

Excess Area=15 to 20%

Binding Edge—A